416

VOLUME 80 SEPARATE No. 416

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

MARCH, 1954

SOIL PROBLEMS IN THE SOUTHERN PIEDMONT REGION

by George F. Sowers, A.M. ASCE

Presented at
Atlanta Convention
February 15-19, 1954

SOIL MECHANICS AND FOUNDATIONS
DIVISION

{Discussion open until July 1, 1954}

Copyright 1954 by the AMERICAN SOCIETY OF CIVIL ENGINEERS
Printed in the United States of America

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

THIS PAPER

--represents an effort of the Society to deliver technical data direct from the author to the reader with the greatest possible speed. To this end, it has had none of the usual editing required in more formal publication procedures.

Readers are invited to submit discussion applying to current papers. For this paper the final closing dead line appears on the front cover.

Those who are planning papers or discussions for "Proceedings" will expedite Division and Committee action measurably by first studying the printed directions for the preparation of ASCE technical papers. For free copies of these directions—describing style, content, and format—address the Manager, Technical Publications, ASCE.

Reprints from this publication may be made on condition that the full title of paper, name of author, page reference, and date of publication by the Society are given.

The Society is not responsible for any statement made or opinion expressed in its publications.

This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

SOIL PROBLEMS IN THE SOUTHERN PIEDMONT REGION

George F. Sowers,[1] A. M. ASCE

INTRODUCTION

The Southern Piedmont Region, Fig. 1, is a broad strip extending from central Alabama across Georgia, the Carolinas, and Virginia, and tapering out to an end in the vicinity of Baltimore and Philadelphia. Within it lie some of the most important manufacturing and commercial centers of the South, and its many desirable features make it attractive for the establishment of many new industries. The soils of this region are different from the more commonly encountered sedimentary sands, silts, and clays, This fact gives rise to challenging engineering problems whose solutions sometimes contradict empirical knowledge based on past experience in other areas. It is the writer's purpose to discuss the engineering features of these soils and point out some of the problems which arise from their peculiar nature.

Geography of the Region

The western boundary of the Piedmont is the Blue Ridge range of the Appalachian Mountains. The eastern boundary is formed by the Coastal Plain sediments which overlap it in an irregular contact known as the Fall Line. The width of the area varies from about 125 miles in North Carolina and about 100 miles in Georgia, to 30 or 40 miles in Maryland. The length is over 800 miles.

Between the boundaries, the general topography is that of a plain which has been eroded into broadly rolling hills. At the base of the mountains, the ground surface lies from 1000 to 1200 ft. above sea level, and it slopes gently to the southeast to an elevation of from 400 to 500 ft. at the Fall Line. The streams cut through the area in deep, relatively narrow V-shaped valleys characterized by rather shallow water depths and occasional shoals.

The hill tops tend to form continuous flat ridges which are the location of the major highway and railroad lines. This is in contrast to more mature topography where transportation routes follow the valleys. Most of the cities are also on the ridge tops and are some distance from the nearest rivers. For example, Atlanta was founded as a railroad junction on Hog Mountain ridge, 7 miles from the Chattahoochee River.

Geology of the Region (1, 2, 3)

The entire region is underlain by crystalline rocks which were formed under tremendous heat and pressure. The oldest rocks consist of gneisses and schists which were formed by the metamorphism of ancient sediments

1. Professor of Civil Engineering, Georgia Institute of Technology
Consulting Engineer, Law-Barrow-Agee Laboratories, Inc.

and igneous rocks in the Pre-Cambrian Era. These rocks all show the evidence of heat and pressure in the segregation of their minerals into parallel bands or sheets. These bands, often mistaken for stratification, range from several inches wide in the gneisses to a fraction of an inch in the schists. These bands are swirled and contorted in much the same manner as hot taffy candy is formed into parallel but twisted stripes. The bands are rarely level; instead they are crumpled and twisted with a general tendency to dip in one direction in any one locality.

Intrusions

Into these rocks have been intruded large quantities of igneous rocks such as granite. In many cases the intrusions have been in the form of large masses such as Stone Mountain near Atlanta. This is an oval shaped granite dome, nearly 700 ft. high and about 7 miles in circumference at its base. Only a relatively few of these masses extend above the general level, however. The age of these intrusions is not known, but they are considerably younger than the schists and gneisses. Yet they also are ancient since their structure indicates that they were formed when the present ground surface was covered with many thousand feet of rock.

Still younger intrusions are in the form of narrow dikes of varied composition. Some are granitic in nature and include many varieties of granite and granite pegmatites (coarse crystallizations of quartz, feldspars, and micas). Others are basic (often termed trap rock) and include diabase, amphibolite, and their altered forms. Some of the dikes are as wide as 300 ft., others are as narrow as 1/8 in., but most range from a few inches to 3 or 4 ft. in width. These dikes represent sharp breaks in the characteristics of the country rock and make an accurate estimate of underground conditions at a site very difficult.

Faulting

The entire area is cut with many faults. The major faults dip to the south and southeast and were formed by thrusting of one side over the other. Most, such as the Cartersville Fault can be traced for miles. In addition, there are numerous minor faults cutting the rocks in all directions. In some cases, the faulting has taken place under such intense pressure and heat as to produce a metamorphism in the rock. In most cases, the result of faulting is the development of a zone of sheared or broken rock from a few inches to several miles wide. The gold mines of the Dahlonega region of North Georgia are located in such a shear zone. All of the faults took place long ago and none appear to have been active during historic times. Therefore, earthquake stresses are ordinarily not considered in structural design in this region.

Landscape Development

The present landscape is basically a peneplain formed after ages of continuous erosion. The present rocks at the ground surface are types which could only have formed deep beneath the earths crust. Therefore, several thousand feet of rock have been eroded, carried away, and then re-deposited in the Atlantic Ocean to form what is now the Coastal Plain Region. Scattered over the Piedmont are low "mountains", the remains of harder rocks which resisted erosion. From the air, they appear to be pimples which rise above the relatively level plain. Examples of these are Kennesaw Mountain northwest of Atlanta and, of course, Stone Mountain.

More recently, the peneplain was elevated above its level of development (or sea level was lowered) and the streams of the region given increased erosive power. These quickly cut into the relatively level surface to form the narrow V-shaped valleys that characterize a youthful landscape. The broad continuous level ridges are the result of this same development.

The Soils and Their Formation

Weathering

The climatic and topographic conditions in the Piedmont are well-suited to rapid and deep weathering. The region is humid (annual rainfall is in the neighborhood of 50 in.) and the rainfall is fairly well distributed throughout the year. The temperature is mild, particularly in the southern part which tends to promote chemical action.

The result has been a rapid chemical breakdown or decomposition of the underlying rocks to form residual soils. The flat topography of the surface did not promote erosion of the weathered rock and so the soils accumulated to considerable depths, often as great as 100 ft. in Georgia and the Carolinas. The subsequent elevation of the plain brought about increased erosion, but it has not yet caught up with the weathering. As a result of this postponement of erosion, the greatest depth of soil is at the tops of the hills while the least depth of soil is in the valleys—a reversal of the condition that exists in other regions.

Extent of Weathering - The most important factors affecting the depth of weathering are the composition of the rock and the defects such as faults and flissures. As was pointed out, the rock minerals are segregated in narrow bands, some of which are more resistant to chemical breakdown than others. Furthermore, these bands are seldom level. The result is an irregular depth of weathering as shown in Fig. 2. Dikes aggravate the irregularity. Some are more resistant than the surrounding rock and lead to hard spots while others are less resistant and lead to deeper weathering.

The faults and fissures serve to carry percolating surface water deep into the rock and develop weathered zones far beneath the ground surface. These intensify the irregularity of the surface of the unweathered rock as shown in Fig. 2.

The boundary between soil and rock is not a sharp one. The degree of weathering decreases with increasing depth until the rocks become sound with only occasional zones of partial weathering.

Transported Soils

Erosion of the residual soils has been relatively rapid since the uplift of the region, and it has been increased during the past 100 years by poor farming practices. The products of erosion have been largely carried out of the Piedmont by the rapidly flowing streams. The transported soils of the region, therefore, are limited to the rather narrow flood plains of the larger rivers and small terraces along the smaller streams.

Soil Profile

The typical soil profile in the region consists of three zones. The uppermost is a crust of deep red sandy or silty clay. The intermediate zone consists of micaceous sandy silts and silty sands. These soils tend to be somewhat softer than the overlying clays and present the most serious engineering

problems. The third zone is one of incomplete weatherine and sandy soils that contain lenses of relatively sound rock. This zone grades into the underlying rock. A typical soil profile is shown in Fig. 3. The soil density is indicated by the penetration resistance which is the number of blows of a 140 lb. hammer falling 30 in. required to drive a 1 1/2 in. I. D. split spoon sampler one foot.

Upper Zone

The upper zone represents the most advanced stage of weathering. The soil minerals include angular quartz, small amounts of weathered mica, clay minerals of the kaolinite family, and iron oxides. The degree of weathering is indicated by the deep reds of the completely oxidized iron compounds. A typical grain size curve, Fig. 4, reflects the degree of weathering in the large percentage of fines. The plasticity characteristics are quite variable. Liquid limits range from 30 to 50 with corresponding plasticity indexes from 9 to 25. According to the Public Roads system, these soils are classified as A-6 and A-7; according to the Unified Classification (Casagrande) system, they are largely in the CL or CL-ML groups.

The soils of this zone tend to be quite stiff. There are at least two causes. First, the natural water contents of the soils are usually less than the plastic limits, apparently because of the long hot summers which desiccate the soils and shrink them. Since the clay minerals are of the kaolinite family, they do not tend to absorb water and swell during wet weather. Second, the leaching of the soluable minerals from the surface downward into the lower horizons of the zone develops some cementing and contributes soil hardness.

The thickness of the crust is quite variable. In most cases, it ranges from 3 to 5 ft., but in some areas, it may be as great as 10 ft. Attempts have been made to correlate crust thickness with topography and ground water level, but with little success.

Intermediate Zone

The intermediate zone is by far the most important from the engi- -ing point of view. Most structures are founded on it, the design of cut slopes and of excavation bracing depends on its properties, and it is an important source of borrow materials.

This zone is formed from the incomplete chemical decomposition of the underlying rocks. Since the breakdown is not complete, the soils still reflect many of the characteristics of the original rock. The most striking feature of many of the soils is their banding. This is the segregation of the soil minerals in narrow, more of less parallel stripes which are the same as the banding of the original rock. The soils, therefore, are ordinarily anything but homogeneous. Instead, they are usually layered or laminated (similar to varved clays), but the layering is erratic, often badly contorted, and usually steeply tilted.

The soil minerals are predominately quartz, clay minerals, and mica. The quartz is in the form of angular fragments which represent the irregular crystals in the original rock. The sizes vary from gravel to silt depending on the coarseness of the rock grain, but at any one point they tend to be somewhat uniform. The clay minerals are predominately of the kaolinite family with very low plasticity. Zones of only partially weathered feldspar often occur within the kaolins. Occasionally lenses or pockets of kaolins, 6 to 8 ft. in diameter and 2 to 3 ft. thick, are found in the soils. These were formed from thick masses of feldspar (probably pegmatite dikes) in the original rock. Some

of the trap rock dikes weather into clays of the montmorillonite family. The montmorillonite clays may present serious problems since the dikes are often small enough to defy detection when borings are widely spaced, but yet large enough and extensive enough to cause trouble.

An important characteristic of most of these soils is their mica content. Most of the rocks contain appreciably large amounts of mica. Since micas are not as easily decomposed as the feldspars, the incompletely weathered soils often contain large quantities of unweathered mica. Some soil bands may be nearly 100 percent mica while mica contents of 20 to 30 per cent by weight are not at all uncommon. Most of the mica flakes are small, in the size range of silts and fine sands, but occasionally larger particles are found.

Depending on the original rock, the soils contain oxides of iron as well as oxides of other metals such as manganese. These are found in varying states of oxidation which gives the soils a wide range in colors from deep reds through pinks, purples, oranges, whites, browns, and even greens and blues.

Soil Classification - The grain size curve for typical soils of the intermediate zone, Fig. 4, usually includes a uniform to well-graded curve in the sand sizes plus a long flat curve that extends through the silt and clay sizes. The first part represents the unweathered quartz and micas, and the second part the kaolins.

Liquid limits usually range from 20 to 40. Unfortunately, it is almost impossible to make an accurate determination on these soils since the soft, wet materials slide in the liquid limit cup instead of flowing. Even grooves fail to prevent the difficulty. Plasticity indexes range from 0 to 5.

According to the revised Public Roads Classification, the soils are predominately A-4 although some might classify as A-5. According to the Unified Classification, the soils are ML and SM. All may be described as micaceous sandy silts and micaceous silty sands.

The micro structure of the natural soils is unique. In the original rock, the mineral crystals were intertwined and interlocked. During weathering, the feldspars alter, but the quartz and micas remain unchanged with their grains still interlocking. The basic framework of the soil then consists of a quartz-mica skeleton with kaolins in between. The void ratios of this structure may be quite high. The highest encountered by the writer is nearly 2.0 with typical values ranging from 0.7 to 1.3. The amount of mica appears to influence the void ratio; the greater the mica content, the higher the natural void ratio tends to be.

Partially Weathered Zone

There is no well-defined lower limit of the intermediate zone. Instead the degree of weathering becomes less with increasing depth until unaltered rock is reached. This area of partial weathering is the third zone. It very definitely reflects the different rates of weathering in different rocks. Lenses of hard, slightly weathered rock may overly softer bands of nearly completely kaolinized feldspars and so the soils may consist of alternately hard and soft layers.

The typical grain size curve indicates the lesser weathering. The particles are coarse, with many gravel sized and boulder sized fragments, and contain only small amounts of fines. According to the Public Roads system, they may be A-1, A-2, and A-3; and according to the Unified System, GW, GF, SW, and SF. Most may be described as slightly silty gravelly sands and silty sands.

Engineering Properties of the Soils

The engineering properties of strength, compressibility, permeability, and compacted density reflect the peculiar nature of the soils. The soils are not homogeneous; therefore, their properties vary considerably in any one area. The soils are layered; therefore, their properties are different in different directions. Numerous tests on samples throughout the area have been performed under the author's direction (4 and 5). A discussion of the results points out what may be ordinarily expected.

Strength

Most of the tests have been performed in triaxial shear using samples from 1.4 in. to 2.8 in. in diameter and with lengths twice the diameters. A few direct shear tests have been performed, primarily in making slow (drained) shear tests.

While many tests have been made, the very complex nature of the soils makes it difficult to draw many generalized conclusions. Some observations are warranted as follows:

(1) There appears to be no appreciable difference between the results of triaxial shear and direct shear tests unless the soils contain definite planes of weakness such as bands of weathered mica.

(2) All the soils have strength properties which can be represented by a Mohr envelope that is a straight line that does not pass through the origin. The equation of this line is

$$s = c + \bar{p} \tan \phi$$

where s is the shear strength, c the "cohesion", $\bar{p}$ the normal effective pressure on the failure plane, and ϕ the angle of internal friction.

Cohesion and Internal Friction - The results of quick triaxial shear tests on a wide variety of samples all show similar trends as is shown in Table 1.

TABLE 1 - QUICK TRIAXIAL SHEAR TEST DATA

Void Ratio	Angle of Internal Friction	Cohesion
0.30 - 0.60	28 - 49 deg.	0 - 1000 psf
.61 - .90	26 - 32	0 - 2000
.91 - 1.20	1.5 - 27	300 - 1500
1.21 - 1.50	0 - 22	500 - 1500

A large part of the cohesion appears to be true cohesion or bonding between the soil grains. In the original rock, the particles were all interlocked with definite physical bonds between them. Apparently, not all these bonds are broken during weathering for many of the soils, especially those containing large amounts of mica, show appreciable cohesion. Part of the cohesion appears to be the result of capillary tension since varying the moisture content (without a change in void ratio) will change the cohesion. A few of the denser soils containing large amounts of quartz and some weathered feldspar have little cohesion. It is possible that the swelling associated with the decay of some feldspar breaks the bonds between the rigid quartz grains. In the micaceous soils, however, the resiliency of the structure can absorb some swelling and these soils retain some cohesion.

The internal friction appears to be true friction plus interlocking of the angular quartz and the mica flakes. A few tests were made on remolded, dried soils at the same void ratio as the undisturbed soils. The angles of internal friction were found to be approximately the same as in the undisturbed samples which indicates that the internal friction is not greatly dependent on the soil structure. The soils with the higher void ratios usually contain larger amounts of kaolin which accounts for their smaller angles of internal friction.

A few tests have been made to compare slow (drained) with quick (undrained) shear. The results indicate that the difference is negligible for the soil tested (micaceous silty sand, void ratio 1.4), but may be appreciable for the very silty soils or clayey soils when saturated. Further study of this point is needed.

Effect of Water Pressure - The development of strength by internal friction depends on the grain-to-grain pressure or effective stress. If the total pressure on the soil is p, the effective stress, $\bar{p}$ is given by

$$\bar{p} = p - u$$

where u is the pressure of the water in the soil voids. This equation indicates that an increase in water pressure is associated with a decrease in that part of soil strength due to internal friction, a phenomenon which has been often observed in these soils.

Effect of Vibration - Ordinarily, soils with void ratios greater than 0.9 or 1.0 are assumed to be very sensitive to breakdown or settlement induced by vibration. In these decomposed rocks, however, vibration appears to have no effect. The writer knows of no cases where vibration has been a source of failure. The high mica content seems to make the very loose structures stable and vibration resistant.

Compressibility and Consolidation

Consolidation tests have been largely made in 2 1/2 in. diameter floating ring consolidometers with samples 1 in. thick. The resulting pressure-void ratio curves such as the one shown in Fig. 5 all have the same general shape as those for other undisturbed soils. On semi-logarithmic coordinates, the curves include a relatively flat section at low pressures; a rapidly curving section, and then a steeper straight-line section. The latter is often termed the virgin compressibility curve. The slope of this curve is described by the compression index and is an indication of the compressibility of the soil.

Compression Index - Values of the compression index range from .01 to 1.3. In clay soils, an emperical relationship has been established between the compression index and the liquid limit, (6). The author has found no such relation to exist in the soils of the intermediate and partially weathered zones. The clayey soils of the upper zone show some tendency to follow the trend of other clays, but the variation from the trend is so great that value of the relation is questionable.

A study of the data from tests on 12 sites in North Carolina, Alabama, and Georgia indicate a definite relationship between the void ratio of the undisturbed soil and the compression index. This is shown in Fig. 6.

Pre-Consolidation - All the pressure-void ratio curves show some evidence of pre-consolidation as can be seen from the typical example in Fig. 5. There is no correlation between this load and the present vertical pressure due to the overburden. The value appears to be the highest in the soils having the least degree of weathering. Therefore, this pressure possibly represents the stresses which once existed because of the particle bonds in the original rock.

Time - Rate of Consolidation - All the soils consolidate fairly rapidly. In the partially saturated soils, a large percentage of the consolidation takes place immediately. When saturation is reached, the consolidation proceeds as would be expected from the Terzaghi Theory. The coefficients of consolidation are high in all but the clay soils of the upper zone. Typical values for the intermediate and partially weathered zone range from 0.25 to 3 sq. ft. per day.

Secondary compression is a minor factor in most cases. It probably controls less than ten per cent of the total consolidation.

Permeability

Permeability testing has been conducted in a limited number of undisturbed samples using falling head permeameters. Typical values are given in Table 2.

TABLE 2 - TYPICAL PERMEABILITY COEFFICIENTS

Soil Description	Coefficient of Permeability in Ft. Per Min. Parallel to Banding	Perpendicular to Banding
Micaceous silty sands	$3\text{-}10 \times 10^{-4}$	$1\text{-}3 \times 10^{-4}$
Micaceous sandy silt, Silts	5-3	.2-1

These permeability values are typical of what is ordinarily expected of silty sands and sandy silts.

The test results consistently show a higher permeability when the water flows parallel to the banding than when it flows perpendicular to the banding. The ratio of the parallel to the perpendicular is typically from 2 to 4.

Compaction

As sources of borrow materials, the soils of the upper and intermediate zones are the most important. These materials are low plasticity clays, micaceous sandy silts and micaceous silty sands, none of which are ordinarly adapted to good compaction.

Maximum dry densities, as determined by the Standard Proctor Test, range from 94 pcf to 110 pcf. In general, the greater the percentage of mica and of the kaolins, the lower the maximum density. Optimum moistures typically range from 22 per cent to 15 per cent. The moisture-density curves usually have sharp peaks, indicating that the soils are very sensitive to moisture content.

Surprisingly enough, the engineering properties of the soils are not as bad as might be expected from the low maximum densities so long as the compaction percentages are high. The soils have moderately high angles of internal friction (from 15 to 30 deg.), some cohesion (500 psf to 2000 psf) and are relatively incompressible. There is little tendency of the compacted soils to swell, but 1 to 2 per cent shrinkage can be expected. In order to obtain these qualities, compaction of from 95 per cent to 100 per cent of the Standard Practor maximum is required. When the compaction percentage falls below 95 per cent, the quality of the compacted soil drops very sharply.

Engineering Problems

As might be expected, the region presents many varied problems in soil engineering. The frequent occurrence of poor soils has led to the adoption of very conservative practices in problems involving soils. Unfortunately, the soil conditions are so erratic that even conservatism based on past experience sometimes leads to failure. Each new site presents its unique combination of conditions that requires engineering analysis for economical design. A few of the more important typical problems are described in the following paragraphs.

Footing Foundations

Bearing Capacity - In most cases, the soil bearing capacity (resistance of the soil to shear failure under foundation loads) is not critical. The combination of permanent cohesion with internal friction leads to high values when the bearing capacity is computed by methods such as those proposed by Terzaghi (7) and others (8). A comparison of computed capacities with those measured by load tests on 1 and 2 ft. square plates indicates good agreement for the denser soils when the angles of internal friction are less than about 30 deg. When the void ratios are over 1.0 and the soils have angles of internal friction of more than 30, the measured capacities are lower than those which are computed on the assumption of general soil failure.

The presence of soft bands or lenses complicates the determination of bearing capacity. When these lenses are large, the design capacity is ordinarily based on the poorest soils encountered at the site. When they are small, the foundations are designed to bridge them. For example, 4 ft. dikes of soft weathered chlorite schist cut across a site which was otherwise underlain by a dense partially weathered granite. Heavy machine foundations were designed as if the dike were a slot in the rock.

Ground water pressure, in reducing that portion of strength that results from internal friction, reduces the bearing capacity. In one site at Lawrenceville, Georgia, footings rested on a stratum of micaceous sandy silt. The stratum was sloping and confined between the relatively impervious clay crust above and partially decomposed rock below. Water pressure developed in the stratum after a prolonged wet spell and the soil weakened to the point it could no longer support the foundations.

Effect of the Hard Crust - The high strength of the clayey crust is both a help and a source of trouble. Small, relatively light structures can be founded directly on the crust with safety even though the intermediate soil zone is relatively weak. So long as the zone of shear beneath a footing is within the crust, the high strength of the crust determines the bearing capacity. In this case, the crust forms a natural mat or raft over the softer intermediate zone.

On the other hand, if the crust is thin enough or the footings large enough that their shear zones extend into the softer soil beneath, the bearing capacity will be less than that of the crust. In this case, footings designed on the basis of the capacity of the crust can be in danger of failure. At a site in South Carolina, the soil bearing capacity was estimated from the results of a load test made on a 1 ft. square plate. The full-sized footings 6 ft. wide failed. A subsequent investigation disclosed that the hard clay crust on which the footings rested extended only about 3 ft. below the footing level. The load test shear zone was entirely in the crust and, therefore, high capacity was indicated. The footing shear zone extended well into the softer intermediate soil and failure resulted.

Settlement - Settlements of sufficient magnitude to cause structural damage are not uncommon in the region. Settlement is largely confined to the soils of the intermediate zone since both the clay crust and the partially weathered zone are comparatively incompressible. Because of the banded soil structure, settlements tend to be very irregular. Often the maximum settlement and the maximum differential settlement are the same. For example, three corners of a large brick mill building rested on partially decomposed rock, while the fourth rested on over 60 ft. of micaceous sandy silts. The settlement at that corner was slightly more than 1 in. The result was severe cracking of the structure since the other corners did not settle appreciably. If the other corners had settled similar amounts, probably little damage would have been done.

In some cases, severe settlements have resulted from a lowering of the ground water level. An industrial plant in Gainesville, Georgia, sought to increase its water supply by drilling a well 50 ft. from a 5 story wall-bearing brick mill building. The soil consisted of 60 ft. of micaceous sandy silts (on which the footings rested) underlain by partially weathered gneiss which contains water bearing fissures. The well was cased to a depth of 67 ft. in order to prevent a lowering of the water level in the soil above. Sixteen hours after pumping of the well was begun, the wall adjacent to the well had settled more than 1 in. It developed serious cracks and even began to lean. An investigation showed that the ground water level in the micaceous sandy silt had been lowered more than 12 ft. Apparently the water bearing fissures were connected with the soil above, and so the well casing had little effect. This failure points out the high compressibility of some of the soils and the speed with which consolidation takes place. It also illustrates the importance of the fissures in the rock on foundations.

Deep Foundations

Because the intermediate soil zone is frequently soft and compressible, large structures often require foundations which bear on the partially weathered zone or on sound rock. Depending on the ground water level, this means deep piers (often termed caissons) or piles.

The erratic depth to rock is a serious problem in the construction of either type. In one building site in Atlanta, the depth to sound rock varied from about 10 ft. in the center to about 70 ft. at one corner, only about 120 ft. away. At a building site in Athens, Georgia, the depth to rock at one corner of a footing was 5 ft., and at the opposite corner about 6 ft. away, 15 ft.

Deep piers are constructed easily if the ground water level is below the deepest excavation. The only difficulty in such cases is the removal of the occasional lenses of partially decomposed rock which occur throughout the intermediate zone. Large (up to 60 in. diameter) augers have been used successfully to bore such piers, and, of course, hand excavation is extensively used.

The foundation for the astronomical telescope for the observatory at Agnes Scott College in Decatur, Georgia, presented a problem since no rock could be found in drilling 60 ft. Ordinarily such telescopes are founded on piers to rock so that they remain in alignment permanently, and they are free from the small movements and vibrations associated with soil foundations. The solution was a massive pier, 4 ft. square and 30 ft. deep with the top portion of the pier isolated from the soil and the building.

Pile foundations have been widely used. They receive their support from end bearing on the partially weathered zone and friction in the deepest

portions of the intermediate zone. Ordinarily the piles drive rather easily through the intermediate zone and reach their final depth rather quickly. Occasionally, isolated hard lenses of partially decomposed rock can stop piles above the partially weathered zone. The result could be settlement from the softer soils below. Fortunately, this is a rare occurrence when driving steel or cast-in-place piles, for the hammer weights and pile strengths are sufficient to break all but the thickest layers. Wood piles, however, often hang up on such hard layers and many have been broken in trying to drive them to the proper level.

The most widely used piles for heavy buildings are the cast-in-place. The depths to rock are within the most economical range for these piles and the ease with which the length of many types, such as the Raymond Piles, can be varied makes them particularly adapted to the regional conditions. Precast piles are not so widely used because their lengths cannot be changed easily to meet field conditions.

Steel H piles are occasionally used for heavy loads where it is desired to break through hard lenses of partially decomposed rock. Penetrations of 5 to 15 ft. into the partially weathered zone were secured with H piles in constructing the foundations for a bridge in Atlanta.

Excavations

Deep excavations have presented many serious difficulties in construction in the Piedmont. Most of these difficulties can be traced to inadequate information about underground conditions or lack of understanding of the nature of the soils.

The irregularity of the surface of sound rock is the most frequent problem. Many engineers and contractors try to estimate the elevation or contours of rock from the records of a few widely-spaced borings or even from experience on nearby sites. In one case, the design of a building had to be revised after construction began since a horn of hard rock was found between two borings which indicated no rock within the excavation level. In another case, the construction of a drive-in theater had to be changed because rock was found to be the highest at the point the excavation was to be the deepest. The only answer to this problem is sufficient borings on the site in question.

Ground Water - Ground water is a consistent source of trouble in excavations in the area. Because of the banded structure, the ground water occurs at irregular levels and is often under artesian pressure because of the dip of the bands. When ground water is encountered in excavations, the soils are often extremely soft or even quick due to the reduction of the confining pressure in the presence of water pressure. At one site at Graniteville, South Carolina, a hillside was underlain by sandy silts (kaolins) containing water under pressure. The hillside was stable when the soil was confined beneath the weight of the topsoil and a few feet of sandy clay. When these materials were stripped off before commencing a basement excavation, the entire hillside became a quaking mass of near-quick silt. Well point drainage was necessary before work would be resumed. At a site in Atlanta, the soil in footing excavations made below the ground water level was so quick that the property was eventually abandoned.

Drainage of the silty sands is usually possible with sumps. The presence of occasional bands of coarse quartz serves to accelerate drainage. The finer micaceous sandy silts do not drain very freely. Vacuum well points have proved successful in these materials.

Water standing on the surface of freshly excavated soils in footing excavations will often cause softening to a depth of 2 or 3 inches. The mechanical bond in the kaolinitic and micaceous soils of the intermediate zone appears to weaken when the soils are freshly exposed to the atmosphere. The cause is possibly sudden release of pressure which results in swelling at the exposed surface and a break between the grains. Since the surface has no confining pressure, there is no strength from internal friction. Therefore, the only remaining source of strength is that due to capillary tension or to absorbed water. Water standing on the surface of the soil will eliminate capillary tension in the surface and reduce the viscosity of the absorbed water. The soil then becomes soupy. The remedy is to remove the softened soil carefully and to pour the footings immediately. As soon as the weight of the footing is on the soil, there is no further effect of water other than that due to pressure as previously described.

Excavation Bracing - As previously stated, the soils possess both cohesion and internal friction. As a consequence, open excavations can frequently be made to considerable depths with steep slopes without the necessity for shoring. Most sewer and water line excavations are made without the use of any shoring.

Unfortunately, exposure to the atmosphere without confinement, exposure to water on the surface, and increased water pressure all tend to reduce soil strength. Many failures of unbraced excavations have occurred. In Atlanta alone, such failures have been responsible for at least 12 deaths in the past six years. Nearly all these failures have occurred at more than 1 day and in some cases, several weeks after the excavation was made. In the spring of 1953, for example, a sewer trench caved in and killed two men a day after it was excavated. The immediate cause appears to have been a long rain which reduced the soil strength. In some instances, failures have occurred along the planes of old faults. Therefore, careful attention must be paid to the structure of the soil. Experience based on these failures indicates that all excavations deeper than about 5 ft. require bracing when the excavations are to remain open for more than a few days or when the excavations are made during wet weather.

Considerable additional study will be necessary before the pressures developed by these soils on bracing systems can be evaluated accurately. Further study is also necessary to determine under what conditions the bracing can be safely omitted.

Fills

Extensive filling is necessary on most construction and highway projects in the region because of the rolling topography. Construction of dense fills is difficult because of the nature of the soils. Because of their relatively low cohesion, the bearing capacity of the soils is a function of the width of the loaded area. This means that the smaller the width of the compaction device, the lower the maximum pressure which may be used. The ideal compaction device for these soils should have a wide contact area, an adjustable total load, and an adjustable pressure so that it can fit a variety of materials. The heavy rubber tired rollers fit these requirements and are extensively used. Sheepsfoot rollers are also widely used, but the foot pressures must not be too high otherwise they will punch into the soil instead of compacting it.

Careful control of moisture is essential in the micaceous silts. The soils are very sensitive to excess moisture and cannot be compacted when very

wet. In many instances, filling operations have to be suspended during the winter when the soils remain wet because rainfall is high and evaporation low.

Well compacted fills of the micaceous sandy silts and silty sands make satisfactory foundations for pavements, floors, and even footings of light buildings. Laboratory tests of compacted soils indicate that high percentages of compaction are required to prevent settlement (usually 95 to 100 per cent of the Standard Proctor Maximum), but when those densities are obtained, the fills provide satisfactory support.

When the fills are poorly compacted, the result is excessive settlement. One fill of micaceous sandy silt 20 ft. high compacted to approximately 85 per cent of the Standard Proctor Density settled nearly 18 in. under its own weight. In a large industrial site south of Atlanta, buildings founded on a poorly compacted fill have developed extensive cracks even though the fill was allowed to stand nearly four years before the buildings were constructed. A retaining wall in Atlanta settled several inches even though the fill on which it was placed had been allowed to settle under its own weight for over 30 years.

All fills made of the micaceous sandy silts and silty sands erode badly. Paved gutters and slope protection in the form of sodding or kudzu are necessary to prevent rapid gullying.

Soil Exploration

Because of the erratic nature of the soils in the area, extensive soil exploration is necessary on even the smaller projects. Past experience on adjacent sites is of limited usefulness and can be very misleading. For example, a soil investigation was omitted on the site of a two-story building in Atlanta because nearby three-story buildings resting on spread footings were free from settlement damage. The new structure required underpinning because the soils on which it rested were completely different from those beneath the other buildings.

The most useful method of soil exploration in the region is test boring which employs soil sampling with a 1-1/2 in. I.D. split spoon sampler driven with a 140 lb. hammer falling 30 in. In this process the soil's penetration resistance is measured by the number of hammer blows required to drive the sampler one foot. By relating this penetration resistance empirically to allowable foundation pressure as determined by engineering analysis, a valuable guide to the design of foundations of smaller structures can be developed. The writer has devoted considerable effort to the development of such relationships. In many cases, the allowable soil pressures in the Piedmont region are considerably higher than those possible in regions of sedimentary soils (9) for a given penetration resistance. Considerable additional study is necessary before a family of curves relating allowable foundation pressure to penetration resistance can be published.

The most accurate approach to soil investigations in the area is an engineering analysis of the results of laboratory tests on undisturbed samples. Undisturbed sampling is not particularly difficult and techniques utilized for clays work satisfactorily. In most cases, simple 3 in. thin-walled samplers secure samples of adequate quality for testing.

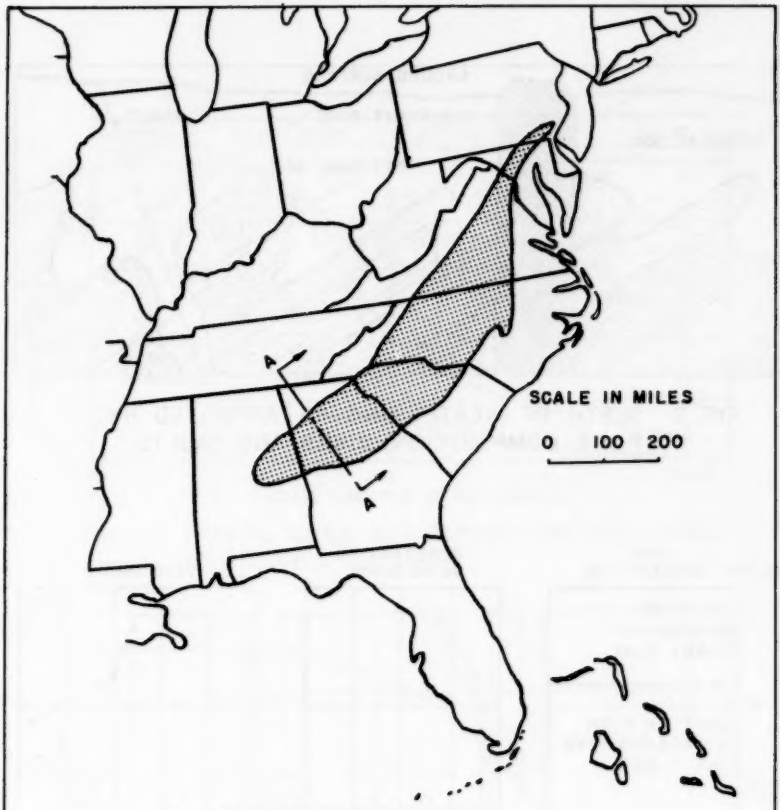
Because of the banded structure of the soil, extensive testing may be required to determine the conditions at each part of a structure. While this is expensive, the increased economy of design is usually well worth the difference. For example, the foundations on one project in North Carolina were first designed on the basis of past experience. An engineering analysis, including undisturbed sampling and laboratory testing, which cost $4000 was able to save over $20,000 in foundation cost compared to the original design.

CONCLUSION

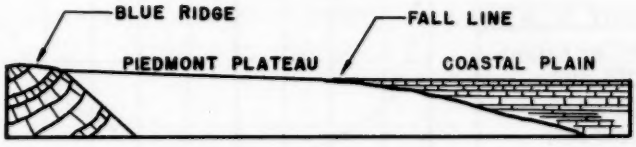
The soils of the Southern Piedmont Region are considerably different from those other areas. The weathering of the underlying gneiss, schist, and granite rocks has created an extremely complex pattern of micaceous sandy silts and micaceous silty sands overlain by a hard crust of red low plasticity clay. The analysis of soil engineering problems in the region cannot be left to guess work or based on past experience in other areas. Instead, the analysis is a challenge to the ingenuity of the engineer. It requires careful investigation of the conditions at each site, a study of the properties of the soils, and then design based upon facts. Above all, it requires an understanding of the geology of the area and the ability to visualize the underground structure and its affect on the proposed construction.

REFERENCES

1. G. W. Crickmay, Geology of the Crystalline Rocks of Georgia, Bulletin 58, Georgia Department of Mines, Mining, and Geology, 1952.
2. Geology of Alabama, Alabama Geologic Survey, Spec. Report 14, 1926.
3. S. M. Herrick and H. E. LeGrand, Geology and Groundwater Resources of the Atlanta Area, Georgia, Bulletin 55, Georgia Department of Mines, Mining, and Geology, 1949.
4. Engineering Reports on file in the Law-Barrow-Agee Laboratories, Atlanta, 1947-1954.
5. Unpublished Reports on Soil Mechanics Research, Soil Mechanics Laboratory, Georgia Institute of Technology, Atlanta.
6. K. Terzaghi and R. B. Peck, Soil Mechanics in Engineering Practice, John Wiley and Sons, Inc., New York, 1948, p. 66.
7. K. Terzaghi, Theoretical Soil Mechanics, John Wiley and Sons, Inc., New York, 1943, p. 118.
8. G. B. Sowers and G. F. Sowers, Introductory Soil Mechanics and Foundations, The Macmillan Company, New York, 1951, p. 114.
9. K. Terzaghi and R. B. Peck, Soil Mechanics in Engineering Practice, John Wiley and Sons, Inc., New York, 1948, p. 430.



A. LOCATION OF THE REGION



B APPROXIMATE SECTION A-A

FIG. 1 - LOCATION AND APPROXIMATE SECTION OF THE PIEDMONT REGION

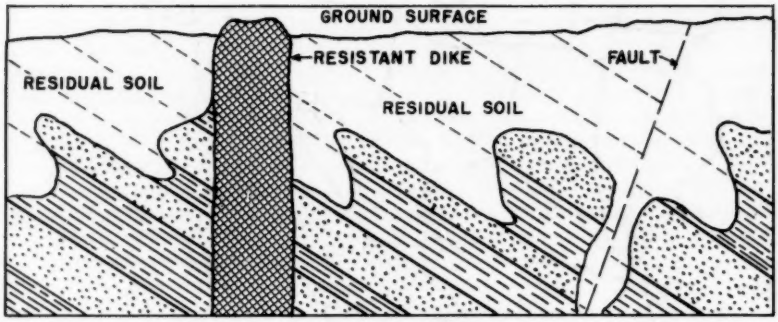


FIG. 2 DEPTH OF WEATHERING AS AFFECTED BY ROCK COMPOSITION, DIKES AND FAULTS.

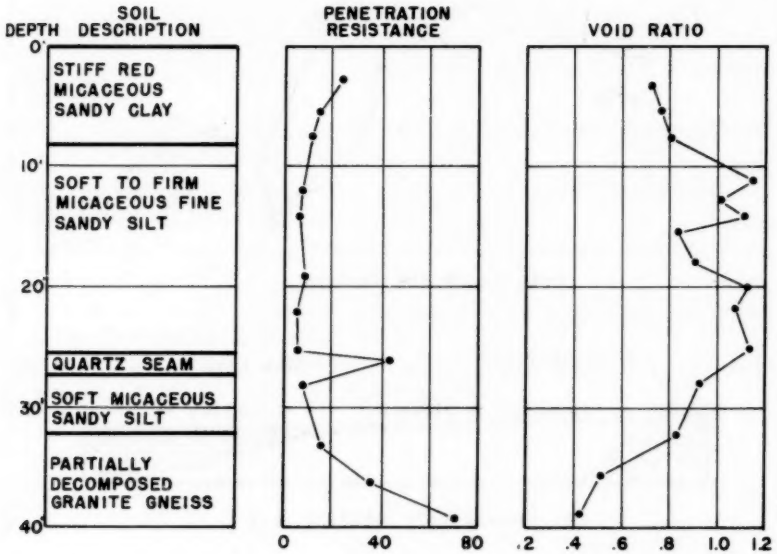


FIG. 3 REPRESENTATIVE SOIL PROFILE FROM A TEST BORING UTILIZING STANDARD PENETRATION AND A BORING 5 FT. AWAY IN WHICH WERE MADE UNDISTURBED SAMPLES. (MADE IN ATLANTA)

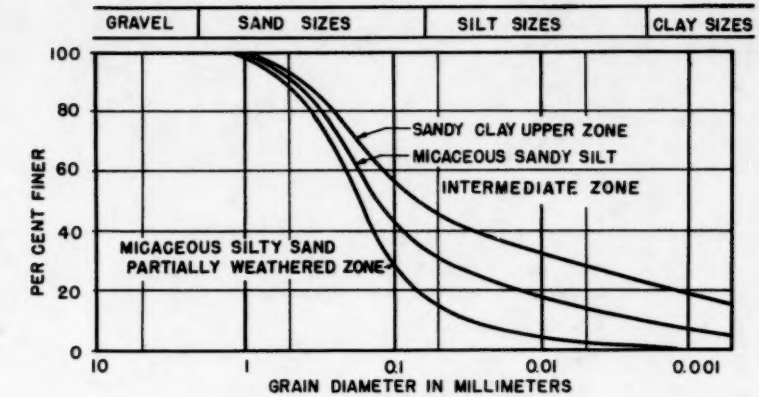


FIG. 4- TYPICAL GRAIN SIZE CURVES FOR THE THREE SOIL ZONES

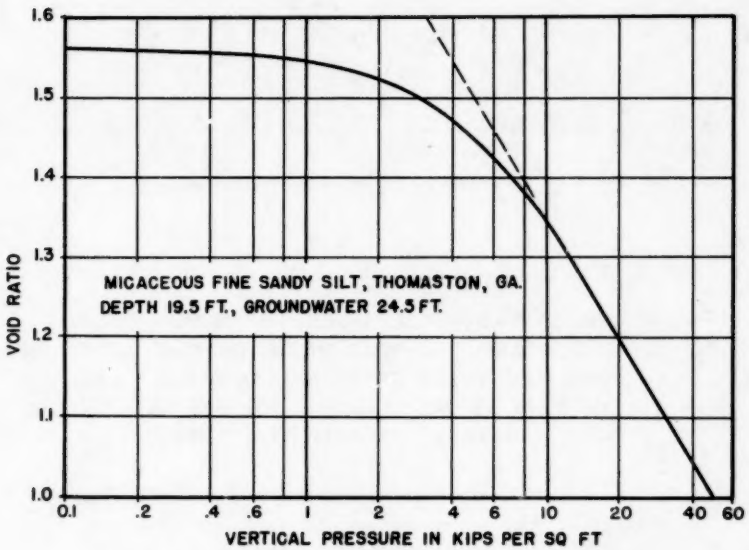


FIG. 5- REPRESENTATIVE PRESSURE- VOID RATIO CURVE FOR A SOIL OF THE INTERMEDIATE ZONE

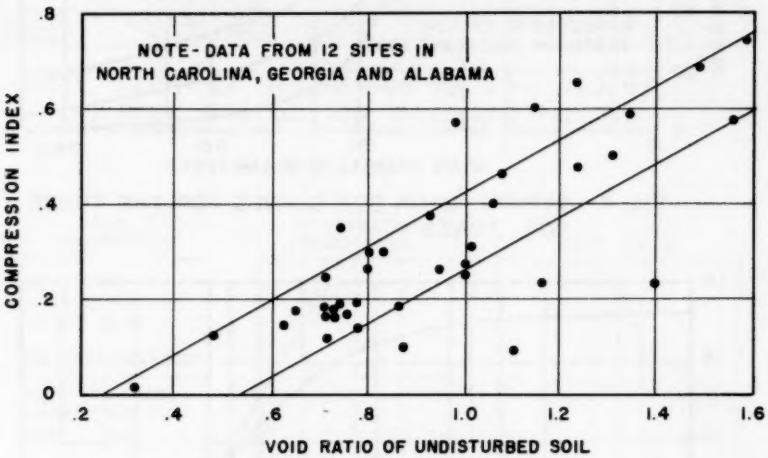


FIG. 6 RELATIONSHIP BETWEEN THE COMPRESSION INDEX AND THE VOID RATIO OF THE UNDISTURBED SOIL FOR THE MICACEOUS SANDY SILT AND MICACEOUS SILTY SANDS OF THE INTERMEDIATE AND PARTIALLY WEATHERED ZONES.